Settlement and Pore Pressure Due to Cyclic Loading

Tassement du Sol Pendant les Chargements Cyclic

H.H. SCHWAB Technical University of Darmstadt, Federal Republic of Germany

SYNOPSIS The cyclic triaxial test as well as the cyclic simple shear test with lateral confinement usually are used to investigate the behaviour of soils subjected to cyclic acting loads. In many cases these testing procedures are unsufficient to be used as base for estimating deformations occuring far away from the liquefied stage. A testing equipment is presented which is well suited for testing soil and rockfill material under controlled static acting horizontal and vertical stresses superimposed by cyclic acting discussed showing the influence of the anisotropic state of the effective stresses and of the drainage conditions on the strains. The behaviour of the soils was investigated after cessating the cycling acting load. Concerning the pore water pressure and the deformations significant results obtained from these tests are reported.

INTRODUCTION

One of the most occuring loading conditions in soil mechanic and foundation engineering are cyclic acting loads. This kind of loads are caused by multiple reasons, they create in soil a large variety of effects both concerning the frequency of load cycles and the stresses and deformations. In most of the cases when vibrations are introduced in subsoil by blasting or machinery foundations e.q., so we have comparatively small stresses and strains on high frequencies. On the other hand cyclic loading effected by earthquakes often produce large stresses and strains within a frequency range of one to five Hz. During a storm wave loaded off shore structures introduce very large stress cycles into subsoil whereby the duration of one load cycle usually takes seconds.

There is a great progress in development of computing models in connection with efficient data processing systems but in the end the calculation of cyclic loading effects induced on soil is insufficient. Especially this holds true in the case that large plastic strains will be mainly determinative particularly in connection with the generation and dissipation of pore water pressure. The necessity to performe model tests as well as to determine the dynamic soil properties in laboratory tests will be of fundamental importance also in future. By this we need a careful consideration in simulating the boundary conditions as lifelike as possible.

In the last two decades many testing equipments and investigating procedures have been evaluated with regard to the above mentioned point of view. In the main the Resonant Column Test (RCT) has succeeded as useful test of determinating the dynamic soil properties for example shear modulus and damping in the range of high frequencies and low shear strains. The Cyclic Triaxial Test (CCT) and the Cyclic Simple Shear Test with lateral Confinement (CSSTC) is commonly used not only for measurement of damping values and shear modulus at medium and large strains but also to evaluate the liquefaction potential of soils. For judgement it is necessary to compare the available testing equipments with regard to their possibilities in simulating the in situ boundary conditions. The most significant in situ conditions will be given in the following. Requirements to testing equipmer and investigating procedures will be summarized.

REQUIREMENTS TO TESTING EQUIPMENTS AND INVESTIGATING PROCEDURES

A soil element in the subsoil usually is loaded by a static acting state of stresses which in most cases is anisotrop that means the horizontal stresses are different from the vertical ones. During cyclic excitation these static acting stresses are superimposed by cyclic acting normal and shear stresses. If pore water exists in the voids these stresses may cause changes in pore water pressure. Whithout regard to the special case of free field condition a soil element initially being in a stable state of equilibrium undergoes vertical a n d horizontal strains in case of cyclic excitation. In consequence the effective stresses are changing caused by a change in pore water pressure. The degree of saturation, relative density, grain size distribution and permeability in connection with drainage conditions as well as the magnitude of static and cyclic acting stresses are determinative to the degree of pore water pressure generation. After cyclic loading commonly the soil element has not reached a stable state of equilibrium immediately. The initial total vertical stress due to overburden pressure and the weight of the structure is acting nearly unchanged also after cessation of cyclic excitation. Whithout regard to the above mentioned special case of free field condition this nearly holds true for the total horizontal stresses. Changes in pore water pressure due to redistribution and dissipation as well seepage pressure leads to additional deformations even after cessation of cyclic loading. This is verified by numerous observations of the behaviour of dams and slopes during and after earthquake induced excitations (e.q. Seed, H.B. 1979; Marcuson, III, W.F. 1979; Ramanujam, N. et al. 1978). So it is necessary to attribute the same attention to the soil behaviour as well after cyclic loading as to the phenomenon occuring during cyclic loading.

The judgement of the investigated soil not only has to include the deformations and failure conditions in case of soil liquefaction caused by excessive pore water pressure generation followed by a significant reduction of the effective shear strength. Even for densely compacted soils the large strains required to mobilize significant resistance may lead to large and unacceptable deformations.

With regard to the simplification that compared with their initial values the total horizontal stresses don't change significantly during cyclic loading and considering only cyclic acting shear stresses the stress condition a soil element is subjected in the subsoil under a loaded area with limited extension or inside of a slope can be represented with a Mohr-Diagram as shown in Fig. 1. The initial state of stresses is represented by point A. Because of the cyclic acting shear stresses and depending on the pore water pressure generated during the excitation the effective stress point moves inside the area ABCD which is signed with PSR. The definition of the dynamic stress path failure line (DK_fL) not necessarily must be done with regard to the state of stresses which represents the shear strength failure condition. Smaller strains which for example are unacceptable for the construction are also applicable as failure criterion.

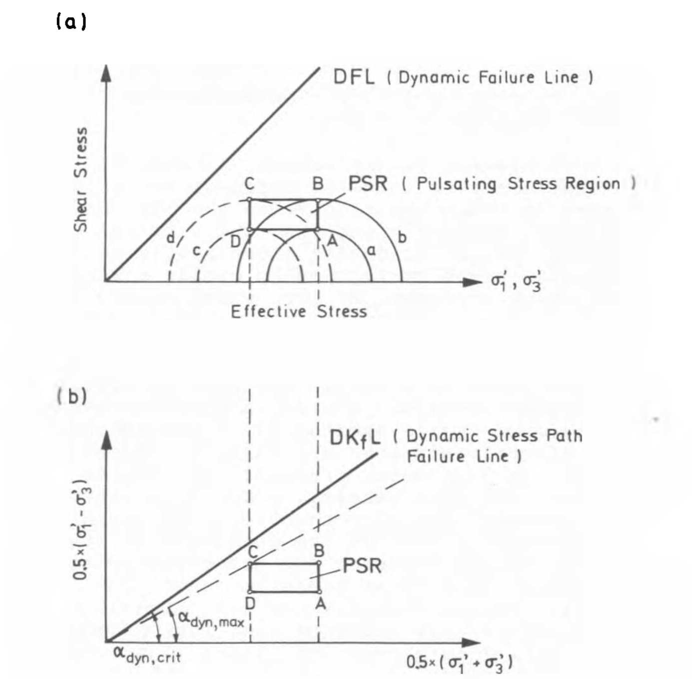

FIG. 1

To investigate the deformation behaviour of soils subjected to cyclic loading the following requirements have to be fulfilled by the testing equipment and the investigating procedure:

- The soil sample investigated in the laboratory test must correspond to the ground conditions in grain size distribution, density and permeability.
- The static effective stresses due to the overburden pressure and the load of structures are to be considered with the ground water level and to be maintained during the test. In gene ral the state of static stresses is anisotrop
- In situ drainage conditions have to be simula ed as lifelike as possible because they are of great importance to the behaviour of soils.
- After cessation of cyclic loading the anisotropic state of stresses should be maintained and the specimen should be allowed to deform without hinderance in vertical as well as in horizontal direction.
- The dynamic acting stresses should correspond to ground condition not only in frequency and amplitude but also in their action as compression and shear stresses.

Due to the simpleness in handling the CTT is wide ly used. Therefore many test data from those are available. The analysation of a grat number of re ported tests from a lot of papers (e.q. Lee, K.L. et al 1967; Seed, H.B. et al 1975; Lee, K.L. 1977 Hedberg, J. 1977 and others) shows that in most o the tests with anisotropically consolidated speci mens the PSR, as defined in Fig. 1, can be represented by the shaded area shown in Fig. 2b and 2c. The effective stress point moves towards the failure line and with further increase of pore water pressure it moves up and down along the very same. In every load cycle the average value of the effective stresses changes in phase with the shear stresses. The direction of the maximum strain values in every moment coincides with the direction of the produced stress increments. The shaded area in Fig. 2a represents the PSR for a CSSTC for example. Due to the complete con-

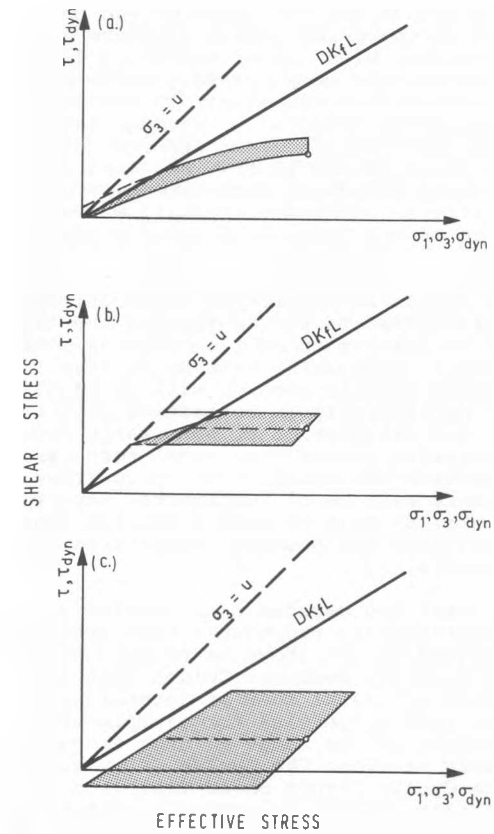

FIG. 2

finement of the lateral deformations by a reinforced rubber membrane the effective lateral stress ratio decreases with increasing pore water pressure. The shear strains measured with the aid of this kind of cyclic simple shear testing equipment cannot be used as a base for a realistic estimation of the deformations which have to be expected. In the same sense this also holds true for the CTT.

TESTING EQUIPMENT USED FOR THE TESTS DESCRIBED HEREIN

To overcome the above mentioned difficulties and to enable us to simulate the in situ boundary conditions as lifelike as possible in the laboratory test a special cyclic simple shear testing device was developed at the Institut for Soil Mechanics and Foundation Engineering of the Technical University at Darmstadt . With the aid of this testing equipment it is possible to load a disk shaped specimen of 2o cm in diameter and 4o cm in height with static acting vertical and horizontal stresses. The specimen is allowed to deform in vertical as well as in horizontal direction. The ground water level can be simulated by a back pressure, the permeability and the thickness of the adjacent soil layers by a drainage control system on the top and the base of the specimen. The vertical seepage from the base to the top of the specimen can be managed by a control circuit whereby different hydraulic gradients and flow velocities can be chosen. On the top and the base of the specimen as well as in the midhight cyclic acting shear stresses can be superimposed to the static acting stresses while drainage conditions and back pressure are kept unchanged. During the processing of the test the static as well as the cyclic acting stresses can be controlled. Volume change, vertical and shear strain as well as the pore water pressure change in the midhight of the specimen can be measured and recorded continuously. Additional information to the testing equipment is given in two previous papers (Breth, H./Schwab, H.H. 1977; Schwab, H.H. 1974)

PROPERTIES OF THE TESTED SOILS

To investigate some problems in the subsoil of several nuclear power plants and of a rock fill dam cyclic shear tests were performed with different soil materials. Not only the determination of the liquefaction potential of the soils were investigated. A main point of interest was the estimation of the deformation which have to be expected during and after earthquake respectively blasting induced excitation. Representative samples were taken from in situ. Fig.3 shows the determined grain size distributions of all investigated soils. The most interesting soil properties are summarized in Table I. The materials No. 1,2 and 4 concern the subsoil of three different nuclear power plants, soil No. 3 was taken from the foundation of aplanned 16o m high rock fill dam. Grain size distributions and soil properties indicate the materials to be unsusceptible against liquefaction.

TABLE I

MATERIAL No.		1	2	3	4
DENSITY OF DRY SOIL	t/m^3	1.666	1.850	1.925	2.093
VOID RATIO	—	0.596	0.459	0.446	0.271
UNIT WEIGHT OF SOLID	t/m^3	2.650	2.700	2.783	2.661
PLASTICITY INDEX	—	0	0.01	0.17	0
DENSITY INDEX	—	0.62	0.75	0.64	0.97

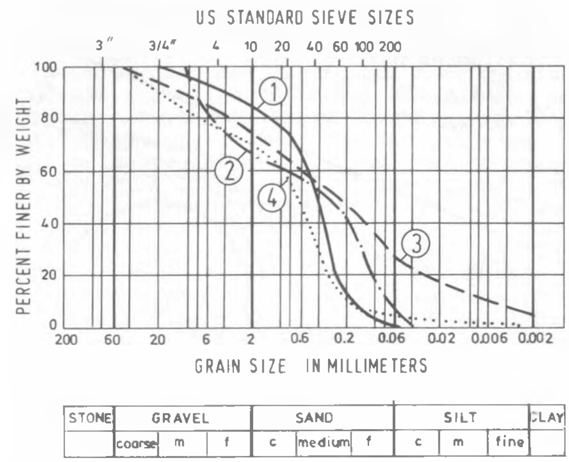

FIG. 3

DYNAMIC SHEAR STRENGTH AND LIMITATION OF STRAINS

To install a DK_f-Line as shown in Fig. 1 the defir ition of a failure criterion has to be given. One possible definition can be given on the base of tl complete reduction of the shear strength of the soil by sufficient large increase of pore water pressure. To analyse the results of the tests witl the material No.1 this criterion was used. The received results have been reported in an earlier paper and shall not be repeated (Breth/Schwab 1977) Compared with the deformations reached in the liquefied stage of the soil much more smaller strains may be unacceptable for structures like dams and nuclear power plants. To define the limiting values for the strains one must take into account the fact that different kinds of strains occur at the same time, for example shear and vertical strain, and that each of them are able to cause unacceptable deformations.

With the material No. 3 test specimens were built in a mould by tamping. The dry density corresponded to that given in Table I. A back pressure was applied sufficient high to get a degree of saturation of 1.o and consolidation took place under horizontal and vertical stresses which differed one from another. The lateral stress ratio was chosen to 0.6 in each test. After consolidation dynamic acting shear stresses with different magnitudes were applied (see Fig. 4). The PSRs measured in each test are shown in Fig. 4. With the assumption that 5 % shear strain as well as 5 % vertical strain are limiting values to keep the deformations small enough for the structure one ends up with the two straight lines which are marked with 2 resp. 3 in Fig. 4. This representation shows that at first the limiting value of the vertical strain is reached if smaller values for the shear stress will be applied and vice versa in the case of large shear stresses the limiting value of the shear strain is firstly reached.

From the representation in Fig. 4 one can read the pore water pressure required in order to move the effective stress point to the defined failure line. No information however is given about the time that means the number of cycles required to reach this state of stresses. Fig. 5 shows the number of cycle which are necessary to cause different values of

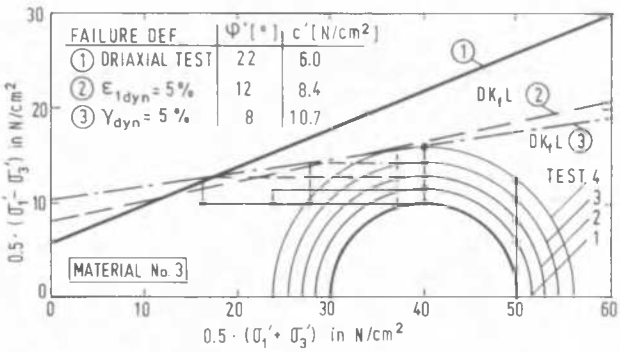

FIG. 4

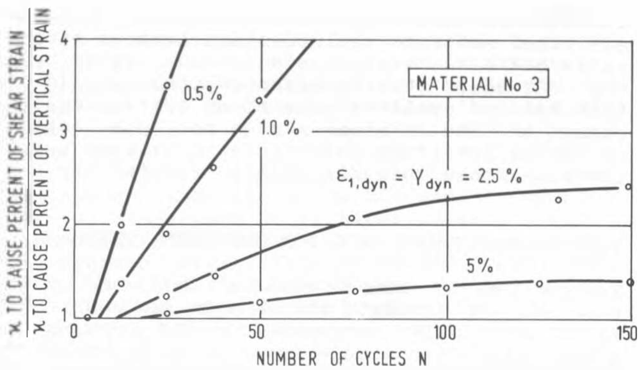

FIG. 6

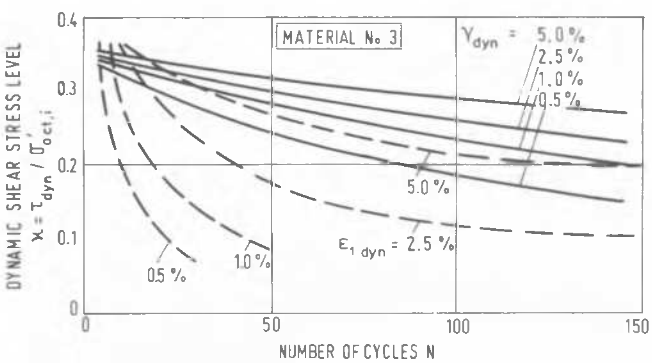

FIG. 5

shear strain for a given value of shear stress level $\varkappa$. To cause 1 % vertical strain within 5o load cycles a dynamic shear stress level of $\varkappa$ =0.08 is necessary. To reach 1 % shear strain within the same number of cycles it needs a dynamic shear stress level of $\varkappa$ = o.27 about 3.4 times of the first value. These relations can be seen more clearly from Fig. 6. The relationship represented in Fig. 6 is determined applying a lateral stress ratio K_i=0.5. Enlarging the K_i-value leads to smaller ratios of $\varkappa$ -shear strain/$\varkappa$ -vertical strain necessary to cause a certain value of deformation and vice versa.

The investigations represented herein show that the vertical strain may reach large values and lead to unacceptable deformations though the shear strain remains small and doesn't indicate any danger. For a given dynamic shear stress level and for a given number of cycles the ratio occuring shear strain to the corresponding vertical strain is significantly influenced by the lateral stress ratio K_i. It is impossible to determine this ratio neither with the aid of CTT nor of CSSTC. In the next chapter the influence of the magnitude of K_i on the shear strain of a sandy silt tested under cyclic loading condition is reported as an example.

ANISOTROPIC STATE OF STRESSES AND SHEAR STRAINS

To calculate the vertical stresses due to the overburden pressure and the load of the structures it can be done more or less successfully introducing some simplifications. In most practical applications we must work with vertical stresses alone because reliable estimates of lateral stresses in situ ar not readily available. To estimate the influence the lateral stresses on the deformation character istics of the soil during cyclic loading in many cases tests only can be done by keeping the verti stresses unchanged and varying the lateral stress ratio K_i. The subsequently described tests were performed by that mode of proceeding.

The soil (material No. 2, see Fig. 3) was compact in the testing equipment to a dry density of 1.85 t/m^3 and saturated afterwards. Back pressure and vertical total stress were kept constant in all tests. The horizontal stress was chosen in such a way that during the consolidation K_i was equal to 0.2 resp. 0.4 resp. 0.6. Tests were performed wit different dynamic shear stress levels. Fig. 7 shc the measured shear strains after 25 cycles corres ponding to the applied shear stresses. One can se that with respect to the range of stresses invest gated the shear strains caused by cyclic acting shear stresses mainly depend on the dynamic shear stress level. That means that a soil element load by a vertical stress corresponding to the overburd pressure and the load of a structure undergoes as larger shear strains as smaller the lateral effec tive stress ratio K_i is. There are indications th the deformation resistance is nearly proportional to the initial average effective stress. This can be seen from the results represented in Fig. 8 wh shows a small scatter of the test data within the investigated range of number of cycles for differ ent K_i-values. Similar results are reported by Ishihara and co-workers obtained from cyclic tors shear tests with a sand (Ishihara et al 1977).

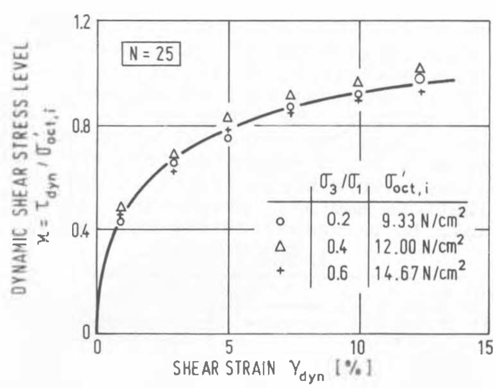

FIG. 7

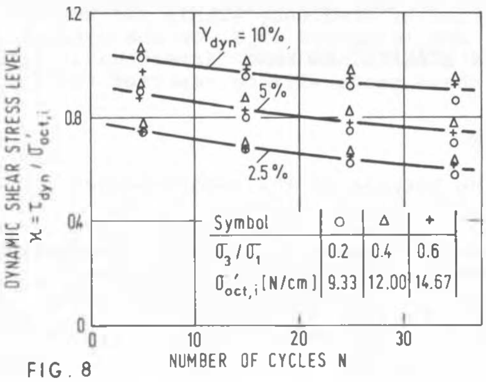

FIG. 8

POST CYCLING BEHAVIOUR

After cessation of the cyclic excitation usually the soil element has not reached a stable condition of equilibrium immediately. Without regard to the above mentioned special case of free field condition one can say that the pore pressure distribution due to cyclic loading is not uniform in the subsoil and therefore ground water seepage is initiated from the beginning of the cyclic excitation. Depending on the geometry and the permeability of the adjacent layers the redistribution of the pore water press-

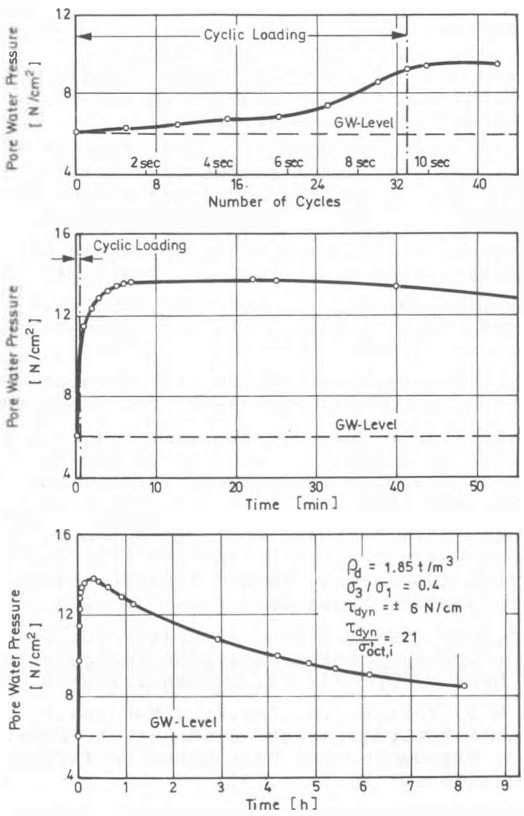

FIG 9

ure and the dissipation of ground water need time and cause additional deformations and change in pore water pressure after cessation of cyclic load ing. Test results describing the Post Cycling Behaviour of the tested material No. 1 were reported previously (Breth/Schwab 1977). As an example for the influence of the Post Cycling Behaviour on the generation and dissipation of pore water pressure with time Fig. 9 represents the measured change in pore water pressure during the test with the sand silt described in the preceding chapter. The test with K_i=0.4 is chosen for representation. The spec men was allowed to drain to the top and the base against the back pressure. The measuring of the pore water pressure was done in the midhight insid the specimen that means that the seepage length wa about 2o cm. The three diagrams arranged one over the other in Fig. 9 represent the same test with different scaling factors for the time axis.
One of the most important results obtained from this representation is the fact that the maximum value for the pore water pressure was measured 22 minutes after the cyclic loading has ended. Eight hours later the pore water pressure hadn't decreased to the initial value of the back pressur During the same time additional deformations with decreasing strain rates could be measured.

This additional deformations have to be taken into account to judge their effect on the settlement an the deformation of structures. Again one must stat that it is also impossible to determine their magn tude neither by CTT nor by CSSTC procedures.

DRAINAGE CONDITIONS AND GENERATION OF PORE PRESSUR

The pore pressure generation mainly is influenced by the possibility of pore water dissipation from the very outset of the cyclic loading. There is an interaction between the pore water pressure and it dissipation in connection with the given drainage conditions, the increase of pore water pressure caused by the load induced decrease of void ratio, the interdependency of change in stresses and last not least the change of the state of the effective stresses by increasing or decreasing pore water pressure. This complex interrelation between all these factors leads to enormous difficulties in performing laboratory tests. Usually these difficulties were overcome by running out the tests wit closed drainage valves. With ragard to the fact that it is possible to liquefy even gravel withou any fines or sand fraction in a cyclic triaxial te provided that the test will be done careful enough one must say running out the tests in undrained condition leads in many cases to conservative and uneconomic results. On the other hand calculations done on the base of drained tests may be unsafe.

To enable us to simulate the in situ drainge conditions in a more realistic manner a drainage control system was developed and test series were carried out with the sand (material No. 1). With the aid of the drainage control system different thicknesses of the layers adjacent to the investigated one were simulated and the volume change, th pore pressure generation vertical and shear strain have been measured. Fig. 1o shows in an extract some of the results obtained in these tests. Curve represents the maximum value of the generated pore water pressure corresponding to the thickness of the layer. Curve B shows the interdependency betwee pore pressure generation value β_i defined by Bjerrum (Bjerrum 1973) and the thickness of the layer. The critical thick-

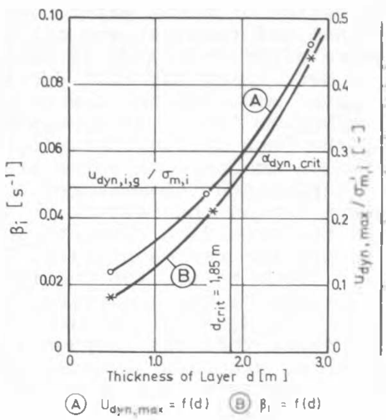

FIG. 10

ness of the layer, that means the thickness which gives resistance to seepage large enough to generate pore pressure sufficient high to cause liquefaction was determined to 1.85 m in this test series (see also Breth/Schwab 1977).

DETERMINATION OF DYNAMIC SOIL PROPERTIES

For the foundation of a nuclear power plant the initially loose sandy gravel in the subsoil was densified by spud vibrators. During compaction about 3o% gravel with grain sizes between 15 and 3o mm was added. Processing by this way a very intensive compaction of the subsoil was achieved. To calculate the effect of a nearby planned blasting on the structure it was necessary to determine the dynamic soil properties. The test specimen should be representativ for the conditions in the subsoil under and beneath the power plant not only due to grain size distribution and density but also due to the state of initial effective stresses. The investigations should be carried out in the frequency range from zero to 2o Hz and the applied shear strains should have values within $1o^{-5}$ to $1o^{+1}$. Because of the size of the testing equipment it was possible to test soil particles with a maximum diameter of 5o mm.

Fig. 11 represents the results achieved from two tests performed under exactly the same conditions. The applied static acting stresses correspond to a depth of 13.5 m below the ground surface. The effective lateral stress ratio for these tests were chosen to 0.5. One of the most significant results indicated by these tests is the fact, that the shear modulus and the damping values is near-

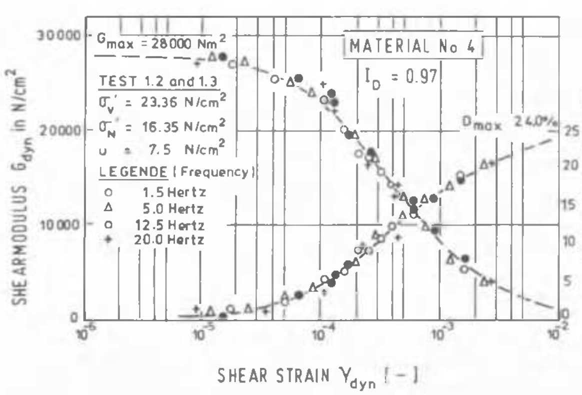

FIG. 11

ly uneffected by frequency within the tested range (Fig.11). The influence of K_i on the dynamic properties of soil is now under investigation. The results of these tests will be reported finally in future.

CONCLUSION

This is the purpose of the report presented to point out some insufficiencies in connection with the widely used testing procedures based on the cyclic triaxial test as well as on the cyclic simple shear test with lateral confinement.. There are certain limitations given by these testing procedures in the possibility to simulate the in situ conditions as lifelike as possible. So for example it is impossible to maintain the initially anisotropic state of static acting stresses during and after cyclic loading. Shear and vertical strains cannot be investigated at the same time, the influence of partial drainage cannot be studied. All these disadvantages can be overcome with the testing prodedure and equipment described herein.

Test results were presented which show the necessarity to overcome all the above mentioned difficulties in the laboratory test. It could be shown, that for example large vertical strains occur which may lead to unacceptable deformations even if no dangerous shear strains will be observed in the same test. The influence of the ratio of the horizontal effective to the vertical effective stress was pointed out to be significant for the magnitude for the occurring shear strains during cyclic loadin

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